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CALIFORNIA METHOD FOR THE STRUCTURAL DESIGN OF FLEXIBLE PAVEMENTS

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SYNOPSIS

For the past several years, the State of California has been using a pavement structural design method based upon test road data and on observed performance of pavement structures. The original formula, containing factors for traffic, supporting power of the soil, and slab strength of the pavement and base layers, has been modified at times as better information became available.

This paper describes not only the design formula but also modifications suggested from a study of the AASHO Test Road data. Correlation with the test track data is also shown.

INTRODUCTION

Soils and granular materials have been used in building construction, for walls, floors, and pavements, for many thousands of years. Obviously, the ancients must have had a great deal of practical knowledge about the use of such materials. When the designers and builders of ballistae, catapults and similar engines of war turned their attention to other forms of construction more precise methods for estimating the potential behavior of materials began to emerge. The need to design stable earthworks was probably most pressing on the military engineers and one of these, Charles Augustin Coulomb, (1736-1806) was among the first to propose a formula by means of which the stability of earthwork embankments might be computed. Nevertheless, in spite of the long history of engineering works involving earthy materials, formulas for calculating the bearing capacity of soils have not been as reliable or perhaps not as well understood as are formulas for bridge members and other structures.

Engineering is a profession which requires an understanding of several sciences and disciplines but which depends primarily upon a knowledge of materials and how the materials will perform or "stand up" under given conditions. The typical engineer has a working knowledge of physics, mechanics, mathematics, and is acquainted with a collection of somewhat inexact numbers and values optimistically referred to as "the strength of materials." The strength concept seems to be reasonable, sound and "common sense." However, it is deceptively simple and can be misleading. A layman knows that a 12- by 12-inch timber beam will sustain a greater load than will a 2- by 4-inch, and can also grasp the idea that a steel beam will support a greater load than a wooden beam of the same dimensions. Carpenters, millwrights, masons and even architects have designed and constructed some fairly elaborate structures without very much in the way of recognizable engineering training. However, while the strength properties of wood, stone or iron may be reasonably well appraised by experience or intuition, this approach has been less successful in estimating the ability of soils and foundations to sustain loads.

A great deal of the difficulty may be ascribed to the lack of means for identifying and measuring the important properties of the materials involved. While the "strength idea" is accepted almost spontaneously and instinctively and presents no serious difficulties when applied to such things as steel, timber, reinforced concrete, et cetera, it does become a little blurred and the image rather fuzzy when one tries to apply this term to the properties of soils. It becomes even more elusive when applied to cohesionless sand and fails completely to describe the properties of liquids such as water.

Webster's dictionary says that strength means:

> "Power to resist force; solidity or toughness; the quality of bodies by which they endure the application of force without breaking or yielding; a measure of the cohesion of material; firmness; coherence; as the strength of a bone, beam, wall, rope, et cetera."

The word strength obviously has many meanings and shadings, and it does not mean the same thing when applied to different materials and circumstances. We may speak of a strong wind or a strong current of water but what we mean is that when either a gas or a liquid is in motion it can

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exert considerable force. A "strong man" may also be able to exert considerable force but he cannot necessarily withstand as much as a "weak woman." At least, women have shown that they often have great powers of resistance! We speak of a strong steel cable or a nylon rope and such strands are strong in the sense of the dictionary definition meaning "cohesion." For most engineering materials, the word strength actually denotes only tensile strength, but materials such as soils can "endure the application of force" and yet possess little or no tensile strength. It, therefore, appears that a more precise general term for the properties in which we are interested is the term "resistance." This term is explicit and may be applied without confusion to a variety of materials. Thus, a strong steel wire or cable requires a considerable force to overcome its resistance to breaking. A column of stone blocks or a dry rubble wall exerts considerable resistance to compressive forces. Even more pertinent to this discussion, the common materials of the earth's crust, rock, sand, gravel, soil or mud, all can be shown to offer measurable degrees of resistance to applied forces. But these materials have little cohesion and hence little or no "strength" unless combined with an artificial binder such as asphalt or portland cement, and even the tensile strength of concrete is not very great compared to steel, for example.

THE PAVEMENT PROBLEM

All pavements, regardless of type, rest upon the materials of the earth's surface, and while there are a few examples of relatively solid rock subgrades, the vast majority of highway pavements are supported by soils or related granular materials having low cohesive strength. Nevertheless, a wide variety of soils have "what it takes" to support pavements if the pavement structure is "properly designed." This means that soils possess some pertinent property other than cohesive strength and this property is easily identified as interparticle friction. The importance of both friction and cohesion were recognized by Coulomb and values for each appear in his formulas.

In order to apply the principles of engineering to the structural design of a pavement, the engineer must know what properties of materials are involved. Lack of reliable tests has been one of the greatest stumbling blocks. Many of the tests which have been applied to soils and paving materials do not provide measures of fundamental properties. For example, if one wishes to measure the tensile strength of steel, a carefully prepared specimen is attached to the jaws of a testing machine and the force required to pull the specimen apart is measured. This is a direct measurement of an important property. If the strength of concrete is involved, a carefully prepared test cylinder or cube is subjected to a direct compression loading. It is important to note that even though steel and concrete are often combined to produce reinforced concrete structures, one rarely attempts to measure the properties in combination. The individual strength properties are evaluated by separate tests. Unfortunately, in the case of soils and other granular materials, a number of test methods are affected by the two distinct properties acting simultaneously.

Many tests provide no means for differentiating between such radically different attributes as friction and the cohesive resistance. While the resistance to deformation or displacement that is due to friction is fairly well defined (if not well measured), the cohesive "strength" or resistance is generally defined as "that portion of the resistance to sliding that is not affected by the pressure." This is a negative definition and differs from the dictionary definition of cohesive strength. In effect then, the soil mechanics definition of cohesion does not define what cohesion is, it merely says what it is not. The other element of confusion arises from the use of such devices as the Mohr circle analysis in which the intercept of the Mohr envelope on the vertical scale is defined as "cohesion." Tests on certain obviously cohesionless materials have shown a definite value for the intercept which would therefore be defined as "cohesion." Finally, a great many have been "thrown off the track" by the substitution of such terms as "shear strength" which by itself is not a property of materials; the total resistance to shear being again composed of variable portions of frictional and cohesive resistance. The resistance due to each of these dissimilar properties combines to produce the total resistance in an endless variety of combinations. The use of tests such as the CBR test, several varieties of direct shear tests, or unconfined compression tests, all tend to reflect or summarize some arbitrary combination of friction and cohesion. The relative proportions depend upon the geometry of the test specimen and speed of loading which usually differ considerably from the conditions on an actual roadway.

Both geologists and agronomists have studied fragmentary stone and the finer decomposition products called "soil" and each have developed classification schemes and names for the numerous varieties of rock, gravel, sand and soil types. These classifications have their uses and have proved helpful to the engineer but none are directly fitted to the engineer's problem. As stated by Dr. Jacob Feld, "an adequate soil classification scheme for engineers should be based upon engineering properties." All this leads up to the point that soil, sand, gravel and other naturally occurring mineral materials possess a number of properties and characteristics and can be variously described according to geologic origin, petrographic classification, grain size, soil texture, mineralogical composition or even in terms of the chemical compounds involved. These classifications may or may not indicate the suitability of the material or the best means of treatment for engineering purposes.

As in the case of all the other sciences concerned with soils, the engineer needs to know what properties are important to him and what determines the ability of the soil to support loads, and having identified these properties he must then know what test methods to use in order to measure them. This is a step which must be made first as no reliable or valid mathematical formula for structural design can be developed unless it includes numerical values to express real and essential properties of the materials involved.

In 1948 a design formula for calculating the thickness of pavements[1] was reported which includes an expression for the measured resistance value of the various soil or granular layers and for the tensile strength or cohesive resistance of all elements composing the pavement structure. The basic data for the relationships developed were derived from a small but full scale project known as the Brighton test track constructed by the California Division of Highways in 1940. For an expenditure of less than $100,000.00, it was possible to construct and operate a test track which included eight different types of base material varying in thickness from 3 inches to 18 inches resting on the same saturated silty clay soil having a CBR value of about 3 or an R-value of approximately 17. The track was subjected to a loaded truck and at the end of the operation it was obvious that the thickness required for the various types of base did not show any consistent relationship to the CBR value or the Stabilometer value for the base material itself, but there was an orderly and consistent trend with the tensile strength of the materials as measured by the Cohesiometer. This test track made it possible to assign tentative values to some of the variables such as the effects of wheel load and repetition. While the underlying soil on the test track was uniform throughout and gave no range of values, some additional check points were obtainable from observations on the State highway system. A few scattered examples where the pavement thickness had been varied over different types of soils made it possible to establish a relationship. The establishment of a scale of values for soil support was greatly simplified by the fact that the thickness of pavement structure required bears a linear relationship to the Resistance Value of the soil as measured in the Stabilometer. There was no opportunity to introduce a variation in tire pressure so the effects of this variable were not established. The formula developed at the time was as follows:

$$T = \frac{(KP\sqrt{a}\ \log r)(P_h/P_v - 0.10)}{\sqrt[5]{c}}$$

where: T = Thickness of cover (base and Pavement) in inches

K = .0175 for best correlation but without any factor of safety. For design purposes it is suggested that K = .02

P_h = transmitted horizontal pressure in the Stabilometer test (#/sq.in.)

P_v = applied vertical pressure in the Stabilometer test (typically 160#/sq.in.)

P = effective tire pressure (#/sq.in.)

a = effective tire area (sq.in.)

r = number of load repetitions

c = tensile strength of the cover material as measured by the Cohesiometer in gms. per sq. in. (approximately = Modulus of Rupture x 45.4)

The above formula was simplified by reducing the effects of load and repition (EWL) to an expression termed the Traffic Index and by reducing the Stabilometer data to a Resistance Value "R". The formula then becomes:

$$T = 0.095 \frac{(\text{Traffic Index})(90\text{-}R)}{\sqrt[5]{\text{cohesion value}}}$$

where: T = required thickness of cover
R = resistance value by Stabilometer

This formula was used for the design of pavements and any discrepancies that become apparent between prediction and performance were noted and modifications in the testing and design procedure were introduced as seemed to be warranted.

Upon the completion of the WASHO test track in Idaho, attempts were made to check the California formula by comparison with the performance on the WASHO test track. Unfortunately, the design of this project was such that only a very few definite points could be established. While the usable data from the WASHO track agreed with the predictions of the formula, they were insufficient to confirm its validity over any substantial range. Fig. 1.

The tremendously larger AASHO test track in Illinois furnishes a great deal more data and gives a much wider range of values for checking a previously established structural design formula. In order to make a comparison between calculated values and test track data, the various materials, basement soils, granular base, subbase and asphaltic pavement were tested and evaluated according to the California procedures. The wheel loads and number of trips were converted through the equivalent wheel load calculation to the traffic index number. With values derived by Laboratory tests of the Illinois materials and calculations for the traffic, it is possible to arrive at a design thickness based on the California formula (1957 Model). The calculated thicknesses may then be compared with the actual thickness reported to be necessary on the test track. The correlation is shown in Fig. 5. The statistical values showing a standard error of estimate of $\pm$ 2.7" and a coefficient of correlation of 0.87 seem to confirm the ability of the California design formula to predict the thickness of pavement required for a wide variety of traffic loads and materials. However, the test track data neither prove nor disprove the applicability of the California formula to other types of soil or granular base materials. The test track pavement structures were supported by only one type of basement soil. Because of this lack of variables on the AASHO project, it is not possible to develop a design formula by using the test track data alone. It will be noted that the statistical type formulas developed by the road test project staff have no terms or identities which permit application to soils differing in properties and ability to support loads from those used on the test track. The test track formula does not identify or indicate means for measuring the properties or physical conditions which account for the performance of the subbases, bases and asphalt pavement types.

THE FACTORS WHICH MUST BE TAKEN INTO ACCOUNT IN A DESIGN FORMULA

A design formula for the structural elements of a pavement should embody all of the important factors which affect the ability of the pavement structure to sustain vehicle loads over a substantial period of years. There have been many formulas proposed. In an article entitled, "Progress Report of Subcommittee on Methods of Measuring Strength of Subgrade Soil--Review of Methods of Design of Flexible Pavements," Professor Kersten[2] listed 22 different formulas. Some of these were based on theoretical concepts, others were completely empirical, and some represented a mixture of the two approaches. The factors which influence the overall performance of a pavement are so numerous and the desirable attributes of a pavement are so diverse that it seems impossible or highly improbable that all of these variables can ever be included in a single formula, or if such a formula were constructed only a highly sophisticated electronic calculator could hope to reach a solution. Even then, a certain allowance would be needed for the inability of construction equipment or methods to do a perfect job.

Fig. 2 is included to show the variables that can affect the performance of an asphalt pavement. It will be noted that at least 30 different items have been identified. However, design formulas rarely need to cover every factor and many of the variables shown on the chart can be ignored or combined into a single element in the formula.

As an example of the simplification which is possible and quite practicable, an adequate structural design might be described as one which produces an economical or efficient pavement that will neither crack nor deform under the assumed traffic during the design life of the pavement. (Guarding against disintegration types of failure is primarily a question of mixture design and quality of materials rather than a structural design problem.) Column 3 of Fig. 2 shows that there are three primary factors; namely, the effects of traffic, the strength of the pavement, and the ability of the foundation to support the load. The primary factors have the following relationship:

$$T = \frac{KDR}{S}$$

where: T = Thickness
K = Constant
D = Destructive Effect of Traffic
R = Resistance Value of Support
S = Strength of Pavement Structure

In order to derive a number to express the load effect, it is necessary to consider Columns 4 and 5 which list some of the sub-divisions which make up the traffic load effect or "the destructive effect of traffic." The principal variables are the total wheel load in contact with the pavement and the number of times this load passes over the pavement. The area of load influence is a factor but the problem has thus far been simplified for highway traffic as the maximum tire pressure on most motor trucks is in the order of 70 or 80 psi for the heavier vehicles. The axle spacing or "the proximity factor" is confined to only two typical configurations; namely, single axles some 15' apart or tandem axles (2 axles within 4'). While the comparative effects of tandem axles versus single axles differ markedly as between flexible pavements and rigid pavements, nevertheless, it is possible to convert these two types of axle spacings to a common denominator for each type of pavement. Examining all of the available data which include the Brighton test track, the Stockton track (constructed by the Corps of Engineers), the WASHO and the AASHO projects, it appears that the relative effects of traffic may be expressed as follows:

<u>For Flexible Pavement Design</u>

$$TI = 1.30\left(\frac{W}{5}\right)^{0.50} r^{0.12}$$

where: TI = Traffic Index
W = Wheel Load in kips
For tandem axles, W = 1.10 individual wheel load
r = Number of load applications

This expression assumes a tire pressure in the range 50 to 100 psi but does not provide for effects of extreme variation in tire pressure as there are insufficient data available to indicate how variation in tire pressure may affect the performance of a road structure. Referring again to Fig. 2, it will be observed that there are a number of factors which compose the overall properties of the pavement. Primarily, there is the question of stiffness* or the resistance to bending. Stiffness of a "flexible" pave-

*The term "stiffness" has been borrowed from a report by L. W. Nijboer and C. van der Poel[3]. Nijboer computes stiffness from the formula:

$$S = \frac{F_p}{X_p} \quad (12)$$

F_p = Force acting on pavement in newtons. Limits of F_p between 10^4 newtons (1 ton) and 2×10^4 newtons (2 tons) respectively.

X_p = Deflection of the pavement in microns.

Therefore, the term "stiffness" bears a simple mathematical relationship to the deflection of the pavement and as used by Nijboer "stiffness" implies the resistance of all components including the pavement, bases, subbases and the underlying soil. For design purposes it seems preferable to us to associate the concept of stiffness with the pavement and base structures alone in which case there will not be a consistent relationship between "stiffness" and "deflection" as the character of the supporting soil will then represent a variable - "resilience."

ment is influenced by the thickness, type and amount of asphalt and by the temperature. This means that an asphalt pavement has a high degree of stiffness during cold weather and it also means that the lower courses provide greater stiffness in warm weather than the same mixture in the surface layer exposed directly to the sun. The stiffness of all materials can be expected to increase with the thickness of the layer but in the case of asphalt pavements the effect is enhanced by the lower temperatures in the bottom courses, especially where the pavement is of substantial depth. Flexibility is more or less the opposite number or complement of stiffness. This is a property not easily measured but it may enable a pavement to survive the flexing over resilient or springy foundations. It is a difficult value to include in a simple design formula. The word "stiffness" is also not entirely applicable or adequate to express the manner in which a pavement structure functions. The concept of "stiffness" is readily visualized in the case of a thick asphalt pavement. It is even more descriptive of a portland cement concrete slab, but a substantial layer of crushed stone or gravel will have the same effect, within the limits of its own resilience, in reducing deflections. Precisely speaking, the term stiffness hardly seems appropriate for a bed of cohesionless material. Nevertheless, in the absence of a better term, a thick layer of sand or gravel may be said to have "stiffness." The question of pavement stability and resistance to water action are properties which fall into the area of mix design and need not ordinarily be considered in a structural design formula.

The process of assigning strength or resistance values to foundation materials must resolve a great many variables due to the wide variety of materials which may be involved. The treated bases and subbases may possess properties similar to that of the pavement layer while granular bases and underlying soils are generally low or completely lacking in tensile strength or cohesive properties. As inferred above, a great deal of the so-called fundamental or theoretical approach to the design problem has focused attention upon the elastic properties but for the most part it is the plastic properties of soils, subbases and granular bases that have caused the most trouble. Again, one must recognize the very dissimilar response of friction and cohesion to most tests or loads.

The Stabilometer furnishes a means for measuring the internal friction of granular materials under load. When solid particles such as stone or sand grains are coated with asphalt or wet clay, a lubrication effect is introduced as soon as a sufficient quantity of the lubricant has been added. Obviously, the amount needed and the effects produced may vary considerably. Rough crushed stone particles are difficult to lubricate while smooth polished gravel and sands will tolerate only small amounts of asphalt or wet clay additions. The problem of stability of asphalt pavements or the ability of granular bases and subbases to support a pavement very largely depends upon the friction or the degree to which the friction has been reduced or lost by lubrication. Thus, the designer of bituminous mixtures or clay-bound stone bases is confronted with the fact that the very materials which are added to increase the cohesion (strength) will also reduce the friction through lubrication whenever sufficient amounts have been added.

When the cohesive effect is provided by a viscous liquid such as asphalt it becomes impossible to summarize the two unlike properties except under some specific condition of load area and speed of loading. Furthermore, it may be shown that the two properties are individually important and each is most effective in certain regions or zones of the pavement structure. A bed of cohesionless crushed stone, gravel or sand will support traffic provided the surface is covered with an adequate thickness of material which does possess some cohesion. A surface treatment or seal coat on a gravel road is an example but to be successful a certain depth of the gravel must have some coherence or cementing action furnished by a soil binder. In contrast, a thin seal coat would be completely ineffective on a bed of clean beach sand. There is ample evidence therefore to show that an adequate pavement structure must provide an upper layer of material having some coherence or tensile strength and the thickness of this layer must increase with increasing wheel loads. Beyond this critical depth, a completely cohesionless gravel or sand will serve quite well and will often prove to be less critical and give more lasting service than will base and subbase layers cemented with natural materials. Natural materials may consist of soil including clay or fines produced by degradation of the aggregate. Fig. 3 is a sketch showing the regions in the pavement structure where cohesion and friction are respectively most influential or important. Fig. 4 is an alignment chart suggesting the depths of pavement and/or cohesive base layer that is required over a completely cohesionless material.

For various magnitudes of wheel loading, the AASHO test road furnishes examples which supplement observations on the performance of actual highways. On Loop No. 2, it will be observed that the thin bituminous surface treatment resting directly on the soil gave a better performance and sustained a greater number of trips before failure than did the same thickness of surface resting on the gravel, yet the soil had a lower CBR, lower R-value and would be considered to be far less adequate by most methods of evaluation thus far developed. Referring to Loop No. 5, the wedge sections 457, 458, 467 and 468 also demonstrate that the failure of the pavement was due to the gravel base as it failed as readily over 15-inches of depth as over 5-inches. Fig. 5 shows the correlation between the thickness computed by the California method (1957 revision)[4] and the actual minimum thickness found to be adequate on the AASHO test track. It appears that the greatest discrepancy between the predictions of the California formula and the actual performance is in the bituminous base sections and it is therefore evident that the assumed cohesive strength value which has been used for California asphalt pavements is not adequate to account for the performance of the thick asphalt sections on the test track.

In evaluating the effect of bituminous bases on performance the wedge sections of the test road provide some information. Table 2.2-9 of AASHO Road Test Report 5 gives information showing the equivalencies in terms of inches of gravel for both the bituminous treated and the cement treated bases. From the AASHO information the equivalencies in the table below were developed:

TABLE I
*TABLE OF EQUIVALENCIES OF TREATED BASES

App. (1000)	Base Type	Equivalency = inches of stone base per inch of treated base							
		Loop 3		Loop 4		***Loop 5		**Loop 6	
		12K-S	24K-T	18K-S	32K-T	22.4K-S	40K-T	30K-S	48K-T
100	CTB			1.8	2.2			1.9	2.0
	BTB	2.9	3.2			2.4	2.3	3.0	2.9
300	CTB			1.7	1.8			1.6	1.6
	BTB	2.8	2.9			2.2	2.3	1.9	2.3
500	CTB			1.7	1.6			1.5	1.5
	BTB	2.7	3.0			2.3	2.4	1.7	2.1
700	CTB			1.6	1.6			1.5	1.5
	BTB	3.1	3.1			2.2	2.4	1.7	1.8
900	CTB			1.6	1.6			1.5	1.5
	BTB	3.3	3.1			2.2	2.4	1.7	1.7
1114	CTB			1.6	1.6			1.5	1.5
	BTB	3.4	3.1			2.2	2.4	1.7	1.7
Avg.	CTB			1.7	1.7	(1.65)	(1.65)	1.6	1.6
	BTB	3.0	3.1			2.3	2.4	2.0	2.1

CTB = Cement Treated Base BTB = Bituminous Treated Base

*Extracted from Table 2.2-9, AASHO Road Test Report 5.

**For Loop 6, four inches of Subbase was replaced by 3.5 inches of Stone Base for comparison purposes.

***Since there was no Stone Base Wedge Section in Loop 5, the average equivalency for CTB (1.65) from Loops 4 and 6 was assumed to be correct for Loop 5 also, and this value was used for comparison with the BTB Sections. The data for Loop 5 are, therefore, interpolations.

The above table would indicate that cement treated bases have an equivalency of 1.65 inches of gravel to 1 inch of base. This agrees quite favorably with California experience as we are currently using a factor of 1.75 to 1.

From the information on bituminous bases, on the other hand, it is apparent that the magnitude of load has a marked effect upon the equivalency of bituminous bases. We suspect that there is also an effect due to depth of layer and the number of repetitions. However, in these latter two cases, it was not possible to isolate the variables by means of the information available. It is possible that one effect offsets the other.

A study of air temperature data at the AASHO Road and corresponding pavement temperature data indicated that an approximate average pavement temperature of about 72° would represent the over-all condition of the test road pavement. Cohesion* (tensile strength) tests were made on AASHO pavement cores tested at various temperatures. The results are shown in Fig. 6. At 72° the cohesion value of the AASHO mix is 5000 grams per lineal inch. The recovered penetration of the asphalt in these cores was 37. To compare a normal California mix using a good crushed California aggregate and asphalt manufactured on the Pacific Coast the remaining series of tests indicated in Fig. 6 were performed. For these California mixes it will be noted that the cohesion at 72° would be only 2000 grams per lineal inch. Most observers would agree that the equivalency of a rigid layer of material, in terms of inches of gravel, should be directly related to its tensile strength and its depth of section. A somewhat different situation exists in the case of bituminous layers for in this case strength not only is related to composition but also to temperature, as is demonstrated in Fig. 6. A bituminous mix varies in temperature from top to bottom, and consequently there is a variation in that portion of its strength which is dependent upon the viscosity of the asphalt binder.

In order to evaluate the property of cohesion, an empirical formula was developed to fit the AASHO conditions. This formula is:

Equivalent Cohesion,

$$C = \text{Cohesion at } 72° \left(\frac{8}{W+2}\right)^{2.5}$$

where: W = applied wheel load in Kips (≥ 6)

Also:

Gravel Equivalency,

$$g.e. = \left(\frac{C}{\text{Cohesion of Gravel}}\right)^{0.2}$$

Fig. 6 indicated that mixes in themselves have widely divergent tensile strength characteristics, and in ordinary highway design problems an equivalency correction for wheel load would not be a simple matter since mixed traffic is involved and the weight of individual axles is rarely known, except on a statistical basis. However, assuming that lightly traveled roads will generally be designed for light loads, and heavy industrial roads will be subjected to heavy loads, a general relationship between equivalency and traffic index can be established.

Fig. 7 is an empirical development from AASHO Test Track data which provides a means of adjusting equivalency for mixes which do not have the tensile strength characteristics of the AASHO asphalt concrete. We feel that these reductions in equivalency are necessary and need to be considered if flexible pavements are to be designed with the assurance of an adequate life. In California, therefore, we are proposing that a series of equivalencies be used that are based upon the predicted traffic.

A table of proposed equivalencies taken from Fig. 7 is shown below. It covers a complete range of traffic currently using California streets and highways.

*Cohesion test is performed by breaking a 2-1/2" x 4" diameter test specimen by bending. Cohesion value = grams per lineal inch to break specimen when the load is applied on a 30" lever arm[6].

TABLE II
TABLE OF PROPOSED EQUIVALENCIES FOR BITUMINOUS MATERIALS

Showing Thickness of Gravel Layer That Would Be Required to Equal 1" of Asphalt Concrete Pavement

Class of Road	Traffic Index Range	Equivalency AASHO Mat'l	Equivalency Calif. Mat'l
Heavy Industrial	10-12	2.0	1.6
Heavy Truck Traffic	9-10	2.2	1.8
Average Highways	7-9	2.4	1.9
Shoulder and Frontage Roads	4.5-7.5	2.8	2.1
Residential Streets	2-5	3.0	2.5

In 1957 the method for calculating traffic index in the California formula[4] was revised. The formula, based upon test road data and experience available at that time, was:

$$TI = 1.35 \left[\left(\frac{W}{5}\right)^5 \text{repetitions}\right]^{0.113} \text{------(1)}$$

where: TI = Traffic Index, a number directly proportional to the required thickness of structural section.

The AASHO Road Test data were reviewed to determine the validity of the exponents in the formula. The number of applications at present serviceability index (psi) = 2.5 was plotted vs the gravel equivalent of the individual sections. These plots on log log paper yielded the following slopes shown in Table III below:

It will be noted from Table III that the use of different base materials results in different deterioration rates due to applications of a given load. However, the estimating of future traffic for purposes of design is, at best, only an approximation. Therefore, we do not feel that refinements in the exponent due to base type is justified until methods of traffic prediction are greatly improved. To encompass all reasonable possibilities, it appears that the exponent of 0.12 would provide a reasonably satisfactory value.

Using the same procedure as above, a tabulation was made for the same test sections in which curves of wheel load vs gravel equivalent were plotted for the indicated number of applications. The slopes are deter-

TABLE III

Loop	Lane	Slope of Curve "Application vs Gravel Equivalent": All Factorial Sections	BTB Wedge	CTB Wedge	Stone Wedge
3	1	0.118	0.088		0.137
	2		0.099		0.111
4	1	0.146		0.100	0.067
	2	0.141		0.127	0.064
5	1	0.093	0.100	0.082	
	2	0.103	0.080	0.103	
6	1	0.097	0.162	0.086	0.046
	2	0.090	0.161	0.099	0.044
Average		0.112	0.115	0.099	0.078

mined for the wheel load exponent. The tabulation is shown below and typical curves are shown in Fig. 8:

Applications of Load	BTB	CTB	Stone
100,000	0.504	0.411	0.563
300,000	0.535	0.476	0.488
500,000	0.595	0.431	0.455
700,000	0.636	0.394	0.380
900,000	0.653	0.349	0.349
1,114,000	0.668	0.359	0.347
Average	0.599	0.403	0.430

Total Average 0.48

In the above table the factorial sections were omitted because we did not have sufficient data to interpolate exact thicknesses for given numbers of repetitions.

The average value of 0.48 is sufficiently close to a theoretical value of 0.50 to justify the use of the latter figure. Using the value of 0.50 our formula for thickness becomes:

$$T = \text{Constant } (W)^{0.50} (r)^{0.12} \text{------}(2)$$

where: T = Thickness
W = Wheel Load
r = repetitions

From equation (2) Wheel Load Constants may be calculated which may be applied to mixed traffic.

And: $$\frac{T_1}{T_2} = \left(\frac{W_1}{T_2}\right)^{0.50} \left(\frac{r1}{r_2}\right)^{0.12}$$

making $T_1 = T_2$; $W_1 = 5000$ lb.; and

r_2 = one repetition of load W_2:

$$r_1 = \left(\frac{W_2}{5}\right)^{4.2}$$ Equivalent 5 kip wheel loads (EWL)

We will call the constants EWL_{62} to differentiate from previous EWL calculations made by the California Division of Highways.

In the paper "Recent Changes in the California Design Method for Structural Sections of Flexible Pavements"[4] the details of using this method to obtain constants applicable to mixed traffic is outlined.

Briefly, the method consists of a statistical sample of traffic as weighed at various loadometer stations throughout the State. The development of the method is shown in the attached Table IV where axle weights at various loadometer stations have been grouped together to show variations within classes of trucks, such as 2, 3, 4, 5 or 6 axle trucks. It will be noted in the table that wheel load factors for the 3, 4, 5 and 6 axle trucks show a variation within a given wheel load group. This is due to allowance for tandem effect. Based on test road data a 10% effect was allowed for each pair of tandems included. The number of tandem vehicles for each class of truck is estimated, using tables published in House Document #91, 1st Session of the 86th Congress. This document contains a large sample of truck combinations and loadings for various geographical areas of the United States. It contains sufficient information to establish the percentage of single and tandem axle combinations for each load group. These percentages were applied to the loadometer tables of the California Division of Highways to determine the average wheel load factor for each class of truck and for each loading.

Table V shows the totals arrived at in Table IV and develops the EWL_{62} constants for computing average daily traffic.

Since in California our traffic counts are reported as the total vehicles in two directions the truck constants developed in the last column of Table V are for these bi-directional counts. It should be pointed out that Table V constants are based on 1959 traffic and that any increase in allowable load limits will result in higher constants. These constants multiplied by the estimated number of trucks of each axle grouping will total to the design equivalent 5000 pound wheel loads (EWL). Constants could also be determined quite readily for equivalent 9000 pound wheel loads.

The EWL may be converted to traffic index by the formula:

$$TI = 1.30(EWL)^{0.12}$$

A typical traffic index calculation is shown in Appendix I. Those who are familiar with and have used the California method previously will note a substantial reduction in the EWL constants. However, the relation between constants (i.e., the ratio of 2 axle to 5 axle or 3 axle to 6 axle vehicles) has not greatly changed.

It should also be noted that for a given traffic situation the new EWL constants will result in virtually the same Traffic Index. For example, in Appendix I the EWL_{57} would have a Traffic Index of 10.9 while the new 1962 constant will give a Traffic Index of 11.1.

By introducing an expression for an increased tensile strength allowance, coupled with a readjustment of the load and repetition exponents, a better correlation with the test road data is possible as shown in Fig. 9. The improved correlation is measured numerically by the reduction in the standard error of estimate from $\pm$ 2.7 inches to $\pm$ 2.2 inches and the increase in coefficient of correlation from 0.87 to 0.93.

It should also be noted that in Fig. 9 all the error is being placed in the subbase layer. This gives a maximum error of estimate and a minimum coefficient of correlation when such things are evaluated in terms of thickness of section. The reason for this is obvious in that the error between actual and calculated thickness must be determined first in terms of gravel equivalent thickness, then converted to inches of surface, base, and subbase. Subbase, having the lowest equivalency, gives the greatest error. Surface material, having the highest equivalency, will give the lowest error.

An example of how the correlation factors might be changed is illustrated by Fig. 10. This represents the same plot as Fig. 9 except that the difference in gravel equivalent was prorated by thickness of layer to surface, base, and subbase. When this is done the error of estimate $\pm$ 2.2 inches of Fig 9 becomes $\pm$ 1.5 inches and the coefficient of correlation raises to 0.97.

SUMMARY

Figs. 9 and 10 serve to illustrate the influence of the method used to judge the efficiency of a design formula. These figures also show that the thicknesses computed by means of the California formula (based on measured properties of the basement soil, the subbase, base, and surface, also the effects of traffic expressed by the Traffic Index) is in nearly all cases equal to or greater than the thickness indicated in the Serviceability Index of 2.5 on the test track. A similar relationship could be shown for 2.0 or 1.5 Serviceability Index.

This is the only relationship which can be justified, as a design formula should provide a structure stronger than any section known to fail. In other words, no portions are expected to show failure within the design life of the project. It may be argued that this provides too great a factor of safety and that the theoretical thickness, in many cases, would be excessive compared to the depths reported as just adequate on the test track. In judging the validity of a pavement design formula by comparing the calculated thickness with test track data, the following facts must be considered:

1. Every effort was made to secure a high degree of uniformity on the test track, and no such uniformity of performance can be expected on a highway constructed by ordinary methods.

2. Traffic was continued on the test track for a period of only two years. This means that the test track did not undergo the large number of cycles ranging from high to low temperature and from wet to dry which affects the performance of a highway over a period of many years.

3. The asphaltic pavements and bases on the test track were only two years old at the end of the test. Virtually all asphalts harden to some degree and become brittle with age. One could not assume an equally good performance over a long period of time on the average highway.

Taking into account the above considerations, any design formula should be on the conservative side and provide some factor of safety over the thickness and strength of pavement which appeared to be barely adequate on the test track. The primary and important advantages of the California formula are:

1. The California procedure utilizes numerical values derived from physical tests of the basement soil, the subbase, base and pavement.

2. The California method provides a logical means for converting miscellaneous traffic wheel loads to a single number,

the Traffic Index, which number bears a direct linear relationship to the thickness of pavement structure required.

3. The California method has been in use for approximately 13 years and has demonstrated that it can accommodate wide variations in the type of soil, type of base and type of pavement as well as variations in wheel loads and in the number of load repetitions.

* * * * *

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In addition to the many persons who have worked on the California pavement design formula for previous papers, the authors wish to acknowledge the help of several whose combined efforts made this paper possible. The review of the AASHO Road Test data was carried out by Mr. David W. Eckhoff, Mr. Robert O. Watkins, and Mr. Carl R. Sundquist. Correlation tests were performed under the general direction of Mr. Ernest Zube. Mr. Clyde Gates, Mr. Daniel Howe and Mr. Merle Nelson all contributed to the testing program and analysis of results.

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(3) "Phenomena in Asphaltic Road Construction", L. W. Nijboer and C. van der Poel, Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953

(4) "Recent Changes in the California Design Method for Structural Sections of Flexible Pavement", G. B. Sherman Proceedings, First Annual Highway Conference, 1958

(5) Throughout the text references are made to data from the AASHO Road Test. These data are obtained primarily from Report 61E, Pavement Research

(6) California Division of Highways Materials Manual of Testing and Control Procedures, Vol. I.

TABLE IV

Calculations to Determine
Yearly ADT Constants for Truck Groups
Based on 1959 Statewide Loadometer Survey

Axle Group Kips	Wheel Load Kips	2 Axle Trucks			3 Axle Trucks			4 Axle Trucks			5 Axle Trucks			6 Axle Trucks		
		EWL Per Axle	No. Axles	EWL	EWL* Per Axle	No. Axles	EWL	EWL* Per Axle	No. Axles	EWL	EWL* Per Axle	No. Axles	EWL	EWL* Per Axle	No. Axles	EWL
2-8	2	0.02	1939	39	0.02	931	14	0.02	1104	21	0.02	3313	55	0.01	153	3
8-9	4-1/4	0.51	115	59	0.45	241	108	0.48	108	51	0.39	859	331	0.44	31	13
9-10	4-3/4	0.81	77	63	0.72	212	153	0.73	90	65	0.62	492	302	0.73	36	26
10-11	5-1/4	1.23	64	79	1.08	157	170	1.10	53	58	0.93	253	235	1.10	27	29
11-12	5-3/4	1.80	49	88	1.58	105	165	1.51	54	82	1.37	261	357	1.55	19	29
12-13	6-1/4	2.54	45	114	2.25	76	212	2.16	50	108	1.99	290	578	2.22	11	25
13-14	6-3/4	3.52	34	120	3.16	71	224	3.00	56	168	2.93	409	1198	3.07	23	70
14-15	7-1/4	4.75	28	134	4.24	105	444	4.05	64	259	3.97	515	2041	4.15	20	83
15-16	7-3/4	6.3	28	177	5.6	114	641	5.3	55	290	5.2	667	3494	5.5	12	66
16-17	8-1/4	8.2	15	123	7.3	66	483	6.9	53	365	6.8	675	4611	7.2	8	57
17-18	8-3/4	10.5	29	305	9.4	38	358	8.8	39	342	8.7	615	5362	9.1	6	55
18-19	9-1/4	13.2	15	198	11.9	16	190	11.2	26	290	11.0	276	3039	11.5	1	12
19-20	9-3/4	16.5	3	50	14.8	1	15	13.9	9	126	13.8	40	551	-	-	-
20-22	10-1/2	22.6	4	90	20.1	-	-	18.8	4	76	18.6	11	205	-	-	-
22-24	11-1/2	33	1	33	32	-	-	28	1	28	27	9	245	-	-	-
24-26	12-1/2	47	-	-	45	-	-	39	-	-	39	3	117	-	-	-
Total No. Axles			2446			2133			1763			8688			347	
Total EWL				1672			3177			2329			22721			468

*Based upon tandem effect (i.e., one tandem = one single 10% heavier than tandem wheel load)

Percentage of single and tandem axles extracted from House Document No. 91, 86th Congress, 1st Session, March 2, 1959

TABLE V

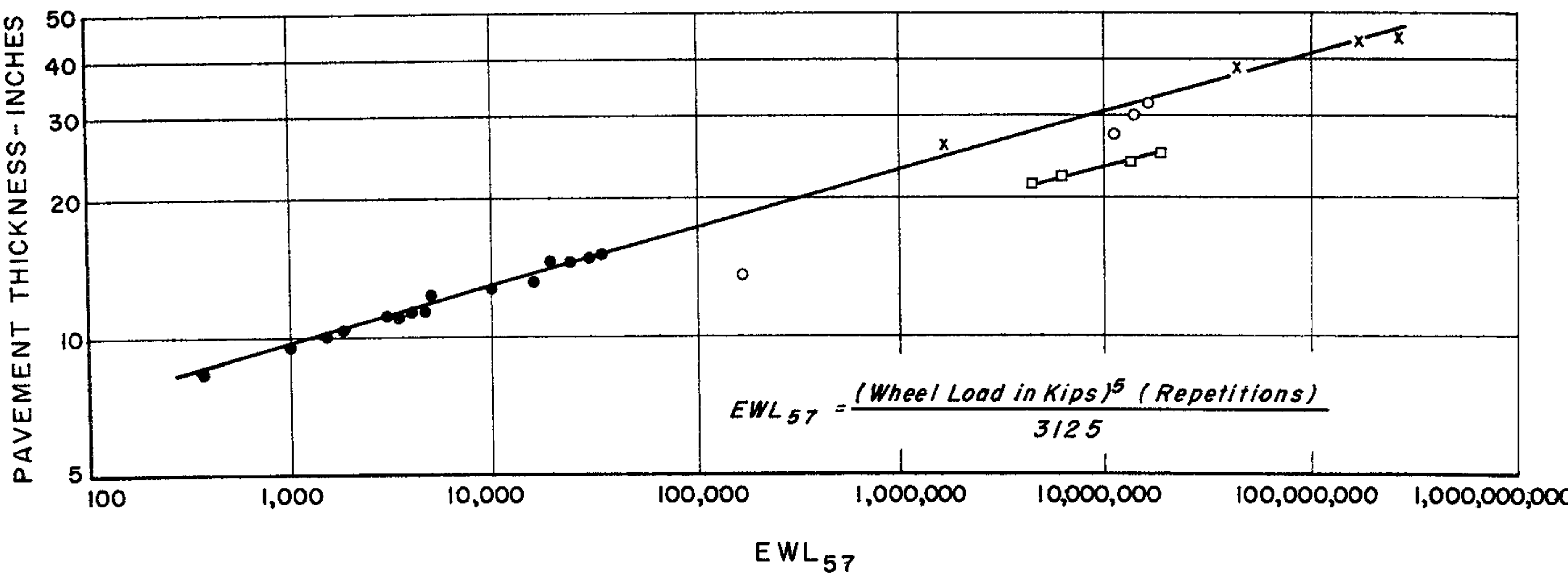
Table of Average Daily Truck Constants for Various Classes of Trucks

No. of Axles Per Truck	Total No. Axles	Total EWL	EWL Per Axle	EWL Per Truck	*EWL for 365 Days	**EWL/year For One Truck in One Direction
2	2446	1672	0.684	1.368	499	250
3	2133	3177	1.489	4.467	1630	815
4	1763	2329	1.321	5.284	1929	965
5	8688	22721	2.615	13.075	4772	2385
6	347	468	1.349	8.094	2954	1475

*Constants when traffic counts cover traffic in one direction only.

**Constants when traffic counts include bi-directional traffic.

CORRELATION OF EWL_{57} WITH PAVEMENT THICKNESS



TEST TRACK DESIGNATION	WHEEL LOAD	TYPE OF SOIL SUPPORT
□ WASHO		17 Avg. CBR Soil
● Brighton	6 KIP	4 Avg. CBR Soil
○ Stockton	25 KIP	5 Avg. CBR Soil
x Stockton	40 KIP	

Figure 1

Analytical Chart To Show The Variables That Must Be Evaluated For The Structural Design of ASPHALT PAVEMENTS

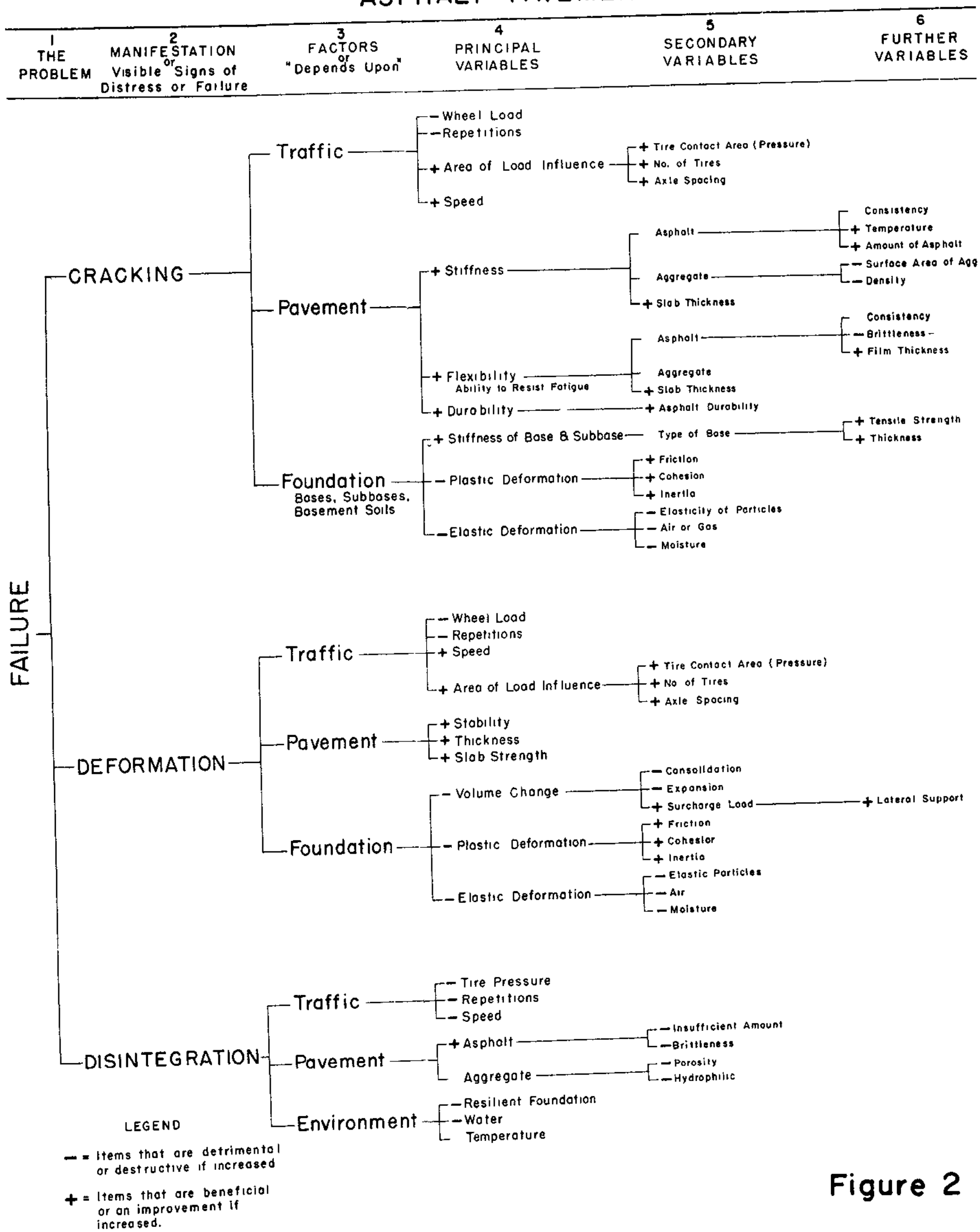


Figure 2

SCHEMATIC REPRESENTATION OF PLASTIC FLOW PHENOMENA IN SOILS SUPPORTING A PAVEMENT

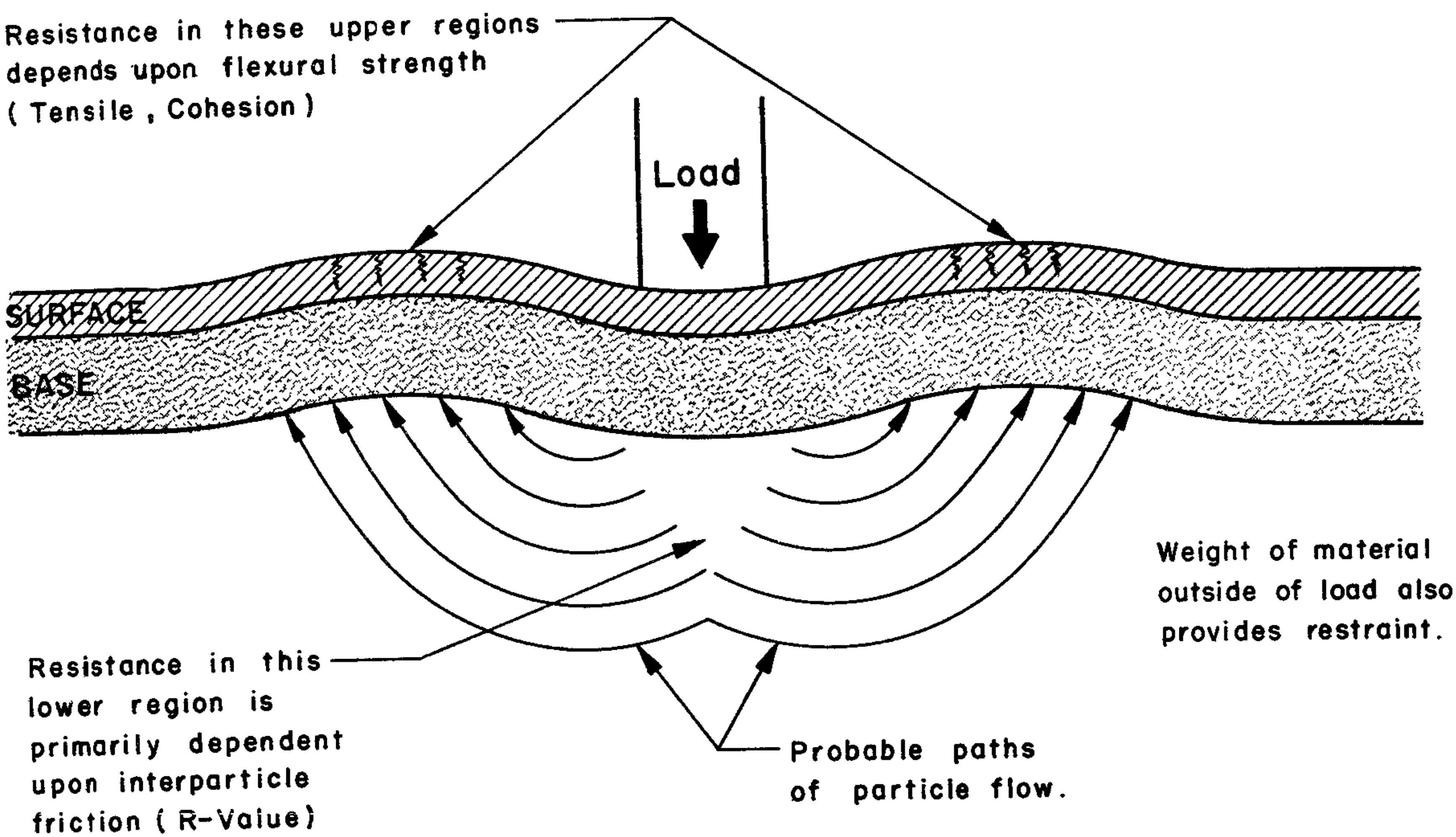


Figure 3

ALIGNMENT CHART TO INDICATE THICKNESS OF COHESIVE LAYER REQUIRED OVER COHESIONLESS SAND OR GRAVEL

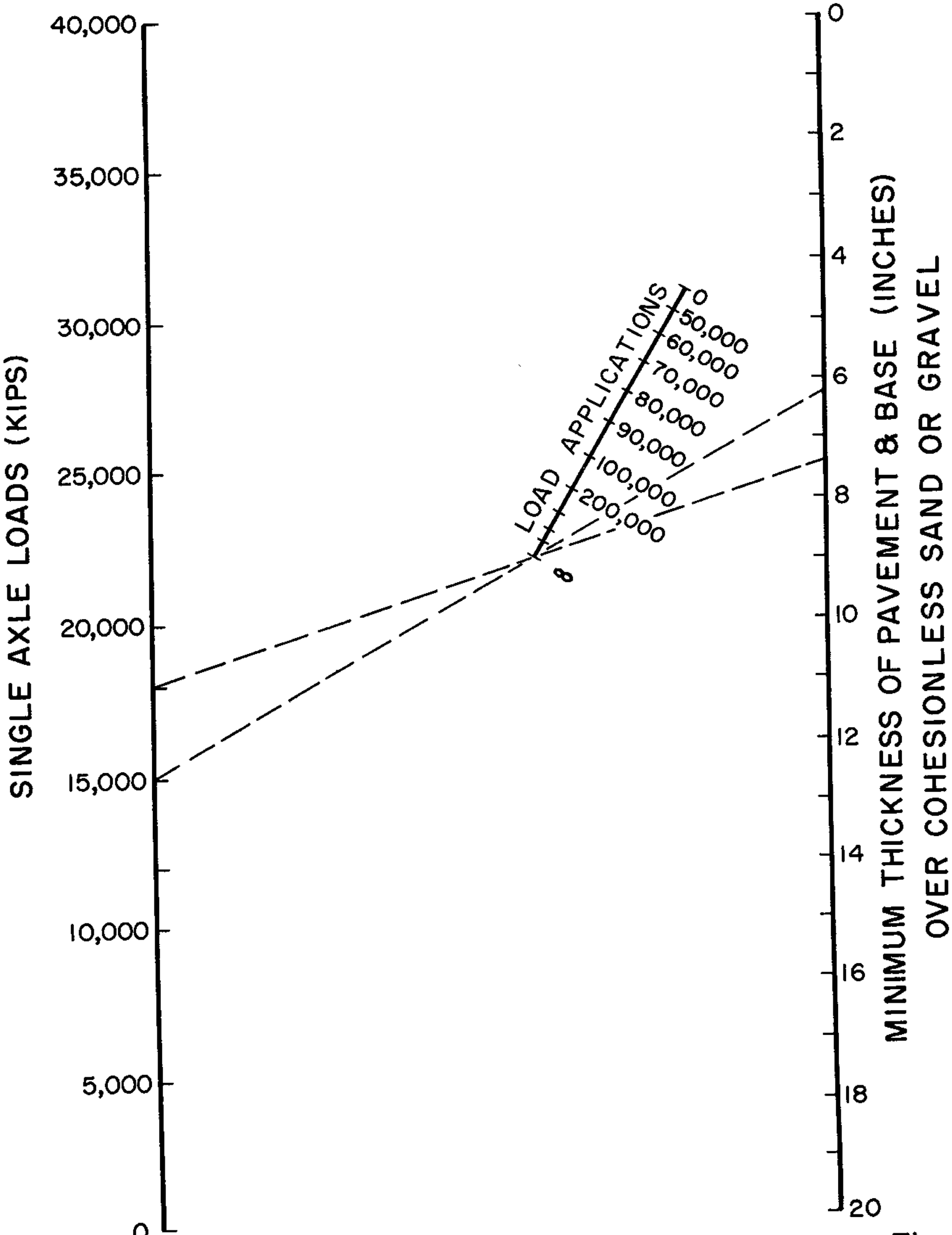


Figure 4

THICKNESS OF TEST SECTIONS AT AASHO TEST TRACK VS. CALCULATED DESIGN THICKNESS USING CALIFORNIA DESIGN EQUATION (1957)

(SECTIONS WHICH FAILED DURING FIRST SPRING THAW ARE OMITTED)

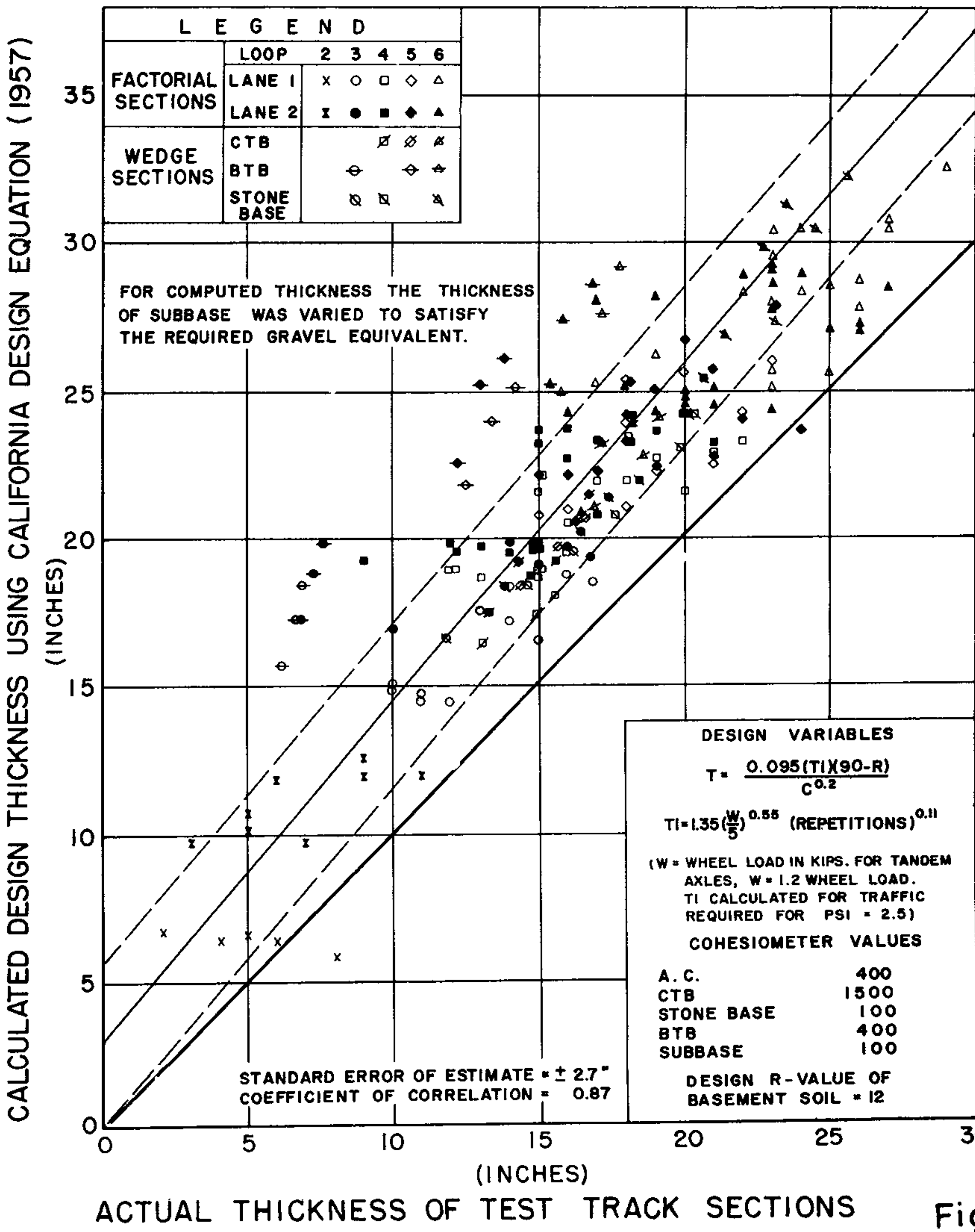


Figure 5

COHESIOMETER VALUES OF BITUMINOUS PAVEMENT AT VARIOUS TEMPERATURES

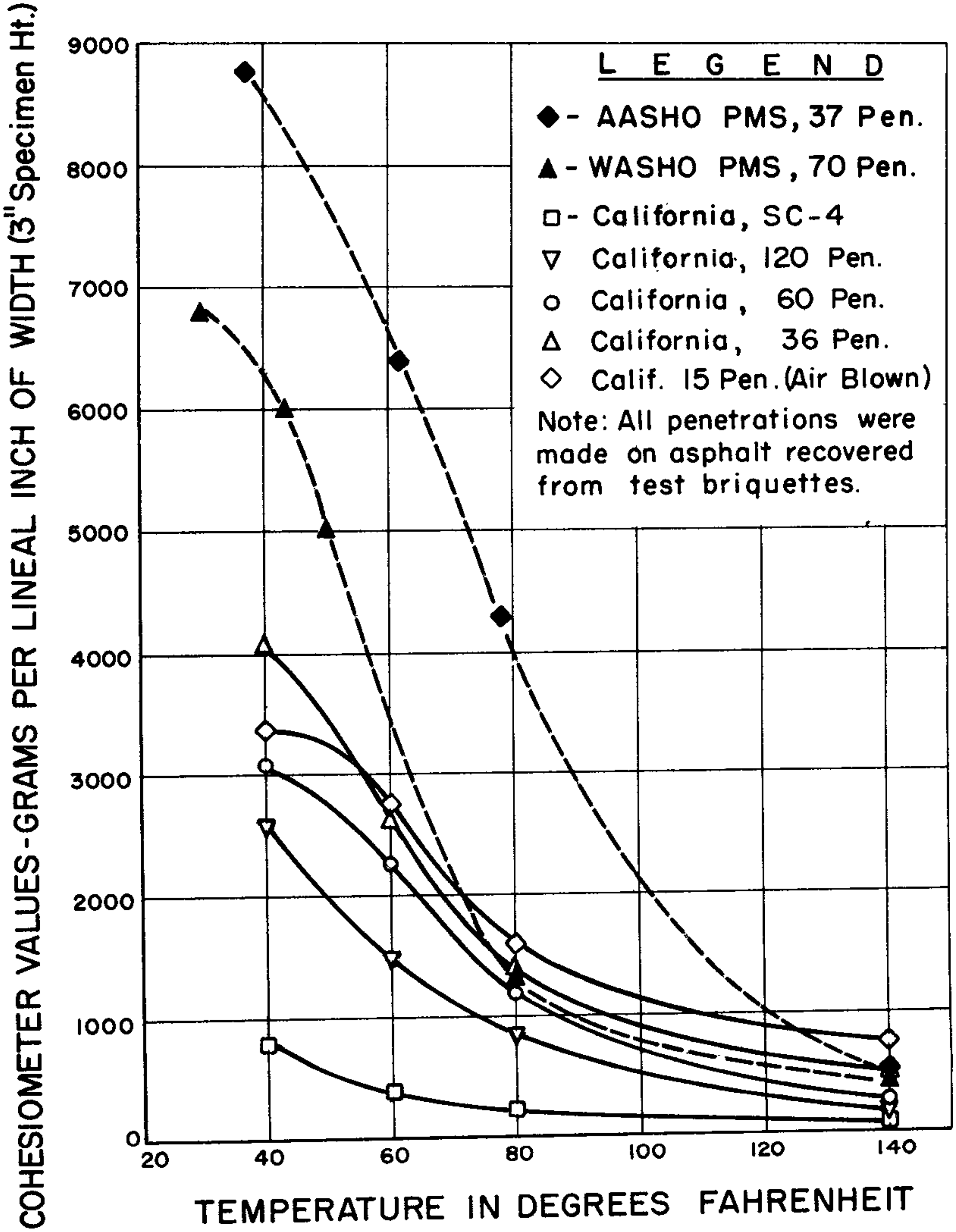


Figure 6

GRAVEL EQUIVALENT OF BITUMINOUS PAVEMENT BASED ON AASHO TEST ROAD ANALYSIS

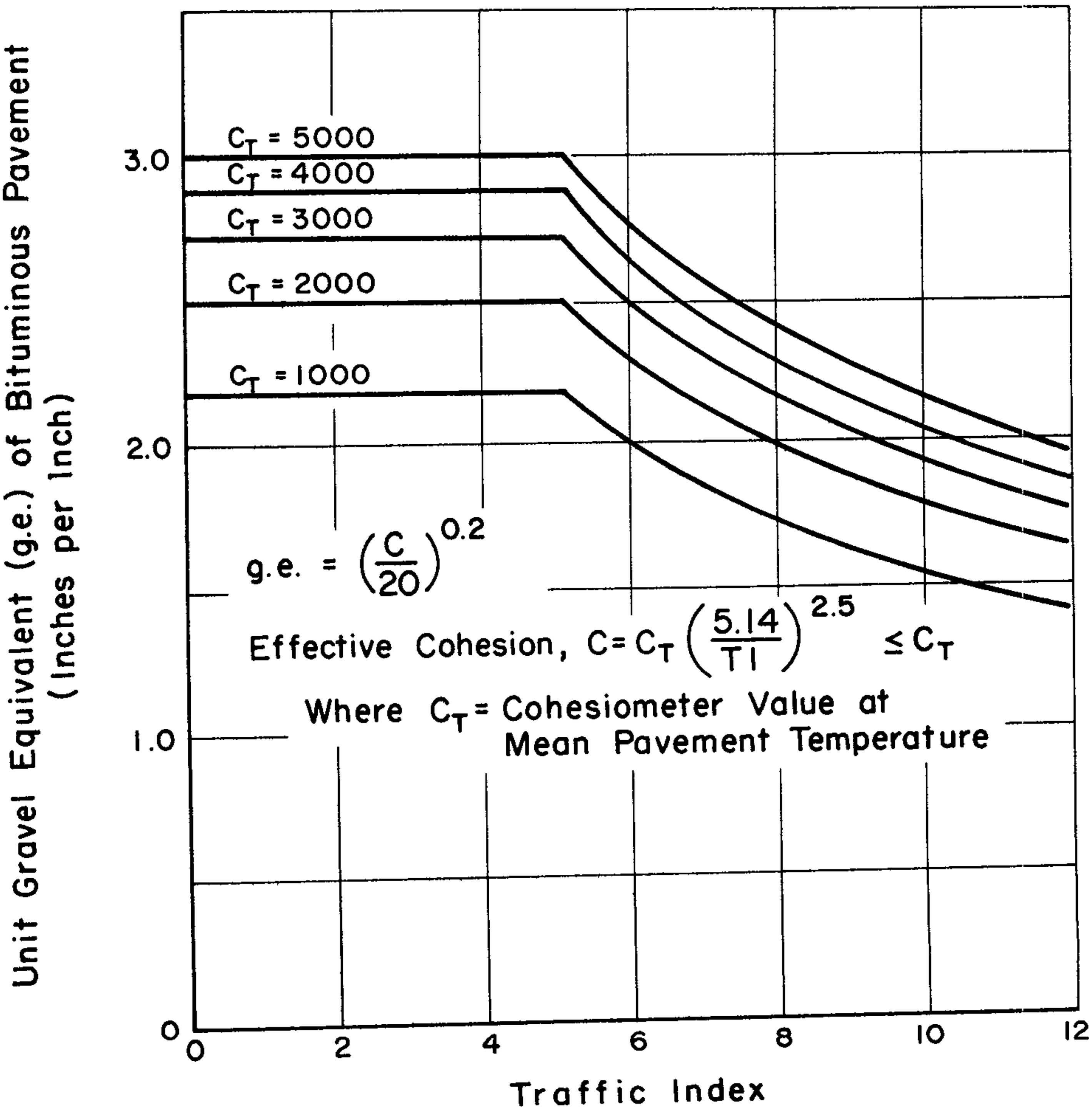


Figure 7

LOG GRAVEL EQUIVALENT OF PAVEMENT SECTION VS. LOG WHEEL LOAD

DATA FROM STONE BASE WEDGE SECTIONS AASHO TEST ROAD

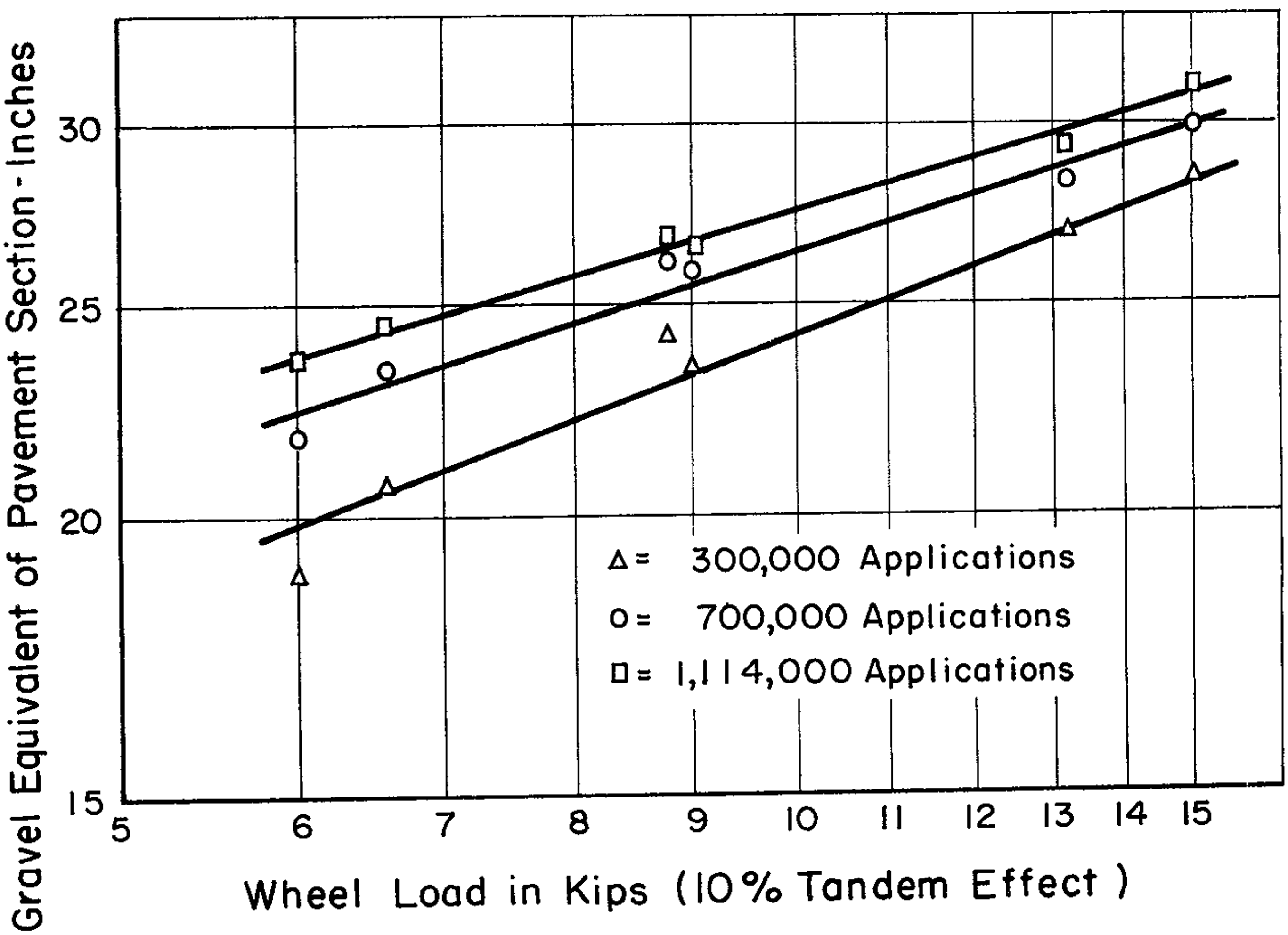


Figure 8

THICKNESS OF TEST SECTIONS AT AASHO TEST TRACK VS. CALCULATED DESIGN THICKNESS USING CALIFORNIA DESIGN EQUATION (1962)

(SECTIONS WHICH FAILED DURING FIRST SPRING THAW ARE OMITTED)

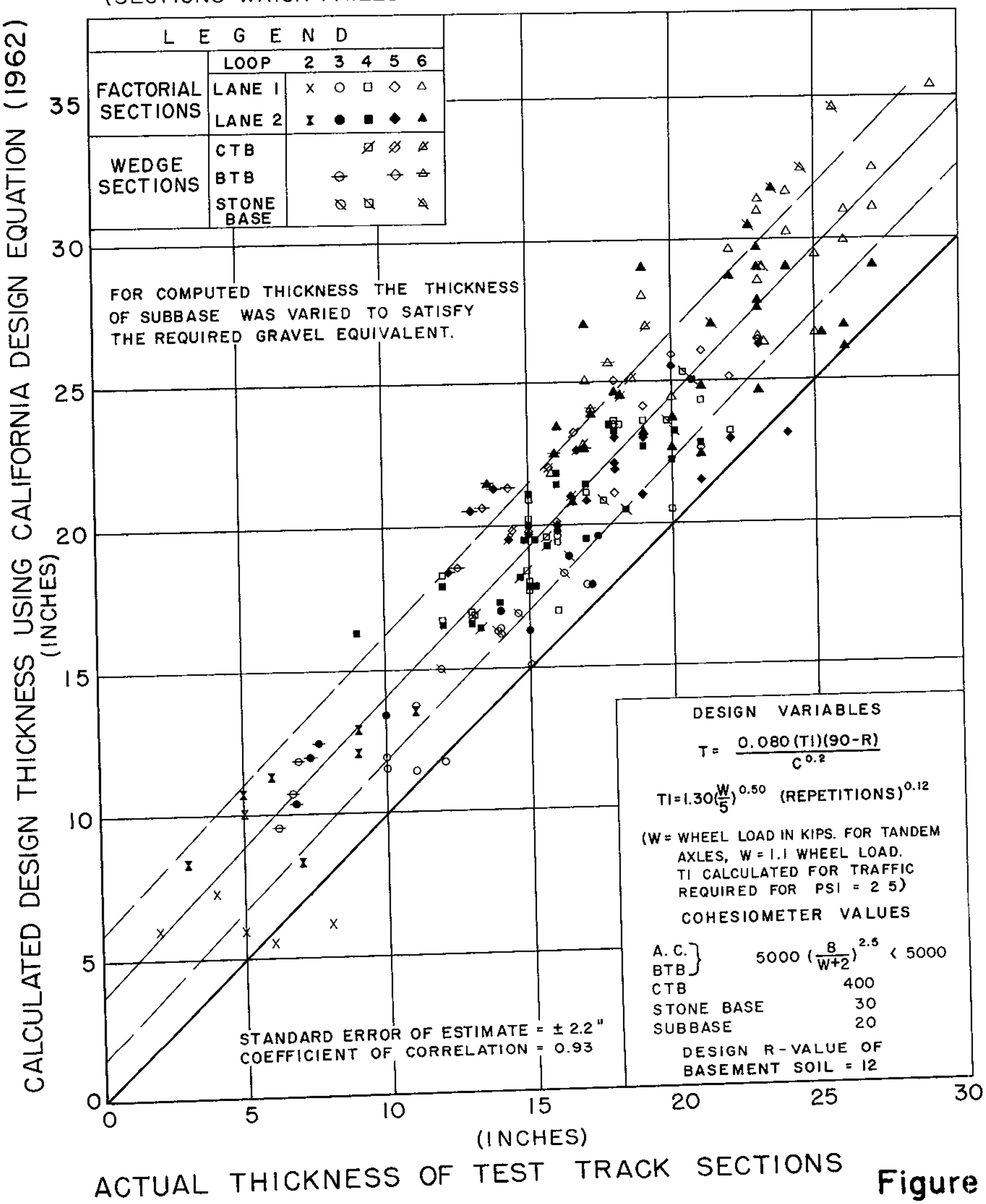


Figure 9

THICKNESS OF TEST SECTIONS AT AASHO TEST TRACK VS. CALCULATED DESIGN THICKNESS USING CALIFORNIA DESIGN EQUATION (1962) (EQUIVALENT SECTIONS)

(SECTIONS WHICH FAILED DURING FIRST SPRING THAW ARE OMITTED)

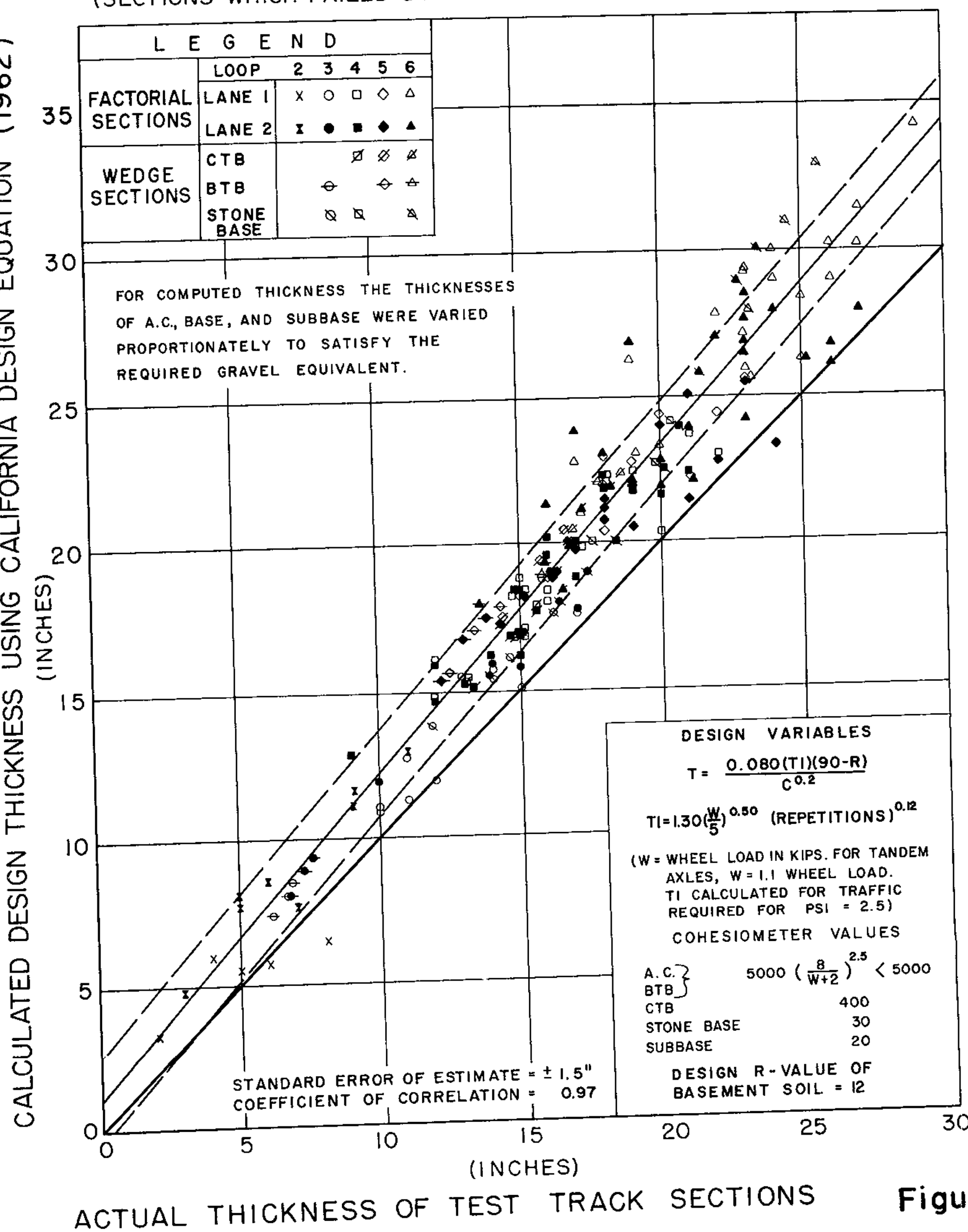


Figure 10

APPENDIX I

TYPICAL EXAMPLE OF PAVEMENT THICKNESS DESIGN

Given the Resistance Value of basement soil = 20, as measured by the Hveem Stabilometer, cohesion of gravel = 20; cohesion of crushed stone base = 30; cohesion of asphalt concrete = 2000, and the average daily truck traffic shown below.

The number of trucks counted in each class is multiplied by the yearly EWL constants listed in Table V to determine the annual EWL.

Truck Class by Axle	*No. of Trucks	X	EWL Yearly Constants	=	Yearly EWL
2	679		250		169,750
3	344		815		280,360
4	295		965		284,675
5	1539		2385		3,670,515
6	113		1475		166,675

Total Annual EWL = 4,571,975

Assuming that in 10 years the traffic will have increased 50%, the average annual design EWL is:

$$\frac{1.0 + 1.5}{2}(4{,}571{,}975) = 5{,}715{,}000 \text{ EWL}$$

The total design EWL for 10 years is:

$$10(5{,}715{,}000) = 57{,}150{,}000 \text{ EWL}$$

Traffic Index is determined from EWL by the following equation:

$$TI = 1.30\ (EWL)^{0.12}$$

For the example above:

$$TI = 1.30\ (57{,}150{,}000)^{0.12}$$

$$= 1.30\left(10^{7.757}\right)^{0.12}$$

$$= 1.30\left(10^{0.931}\right) = 1.30(8.53) = 11.1$$

*2 directional count

PAVEMENT THICKNESS CALCULATION

The required Gravel Equivalent (G.E.) is determined by the equation:

$$G.E. = \frac{0.080(TI)(90-R)}{(\text{Cohesiometer value of gravel})^{0.2}}$$

For the example:

$$G.E. = \frac{0.080(11.1)(90-20)}{(20)^{0.2}} = 34.2''$$

Based on California experience, the minimum thickness of A.C. for a Traffic Index of 11.1 is 7 inches. For a crushed aggregate base, a thickness of 10 inches is typical in present California practice.

The Gravel Equivalents of the A.C. and Base are tabulated below:

Materials	Cohesiometer Value	Gravel Equivalent
A.C.	2000	(7)(1.6)* = 11.2"
Agg. Base	30	(10)(1.1) = 11.0"
	Total	22.2"

The thickness of subbase (g.e. = 1.0) is therefore:

34.2 - 22.2 = 12.0 inches

The total structural section is:

7" A.C.
10" crushed aggregate base
12" subbase

*Refer to Table of Proposed Equivalencies, page 9.